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# Computer design of tunneling with soil freezing

## Le projet par ordinateur de tunnel avec congélation du sol

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SYNOPSYS: For the process of freezing of soils in tunneling a numerical analysis has been developed based on the simultaneous solution of the diffusion equation for the heat transfer, the continuity equation of the water seepage, and the equilibrium equations for continua on the condition of small strains. The unfrozen as well as the frozen soils are treated as hypoelastic bodies obeying Coulomb-Mohr's failure criterion. For homogeneous, isotropic soils a computer program has been elaborated using the finite element method based on Galerkin's method. The results are presented by isochrones of the freezing progress and by time-dependent graphs: isotherms, isolines of shear-stresses, of excess pore-pressures, and of safety coefficients. The behaviour of the frozen zone after the excavation of the tunnel has been analysed by the FINEL computer program.

## INTRODUCTION

Freezing of soil as a mean to increase its strength and stability in underground working or tunneling has been used for more than a century. While the technology of freezing was constantly improved and its costs lowered, its application became even more important. On the other side, the methods of analysis of frozen soil were improved. Up till now, these methods were based on a separate calculation for stress-strain states on the one and on the stability of soil on the other side. Recent numerical methods and the use of computers offer possibilities for the simultaneous solution of diffusion and equilibrium equations taking into account non-linear stress-strain relations including the failure states.

This paper deals with the analysis of the process of freezing of soil. A finite element analysis developed on the basis of Galerkin's method has been introduced. The model for soil is a hypoelastic body which obeys the Mohr-Coulomb failure criterion. The time-dependent heat transmission and stress-strain state in the frozen wall which is surrounded by unfrozen soil has been treated and applied to a plane-strain example in tunneling. In its initial state the soil is treated as a homogeneous, non-viscous half space. Soils of different permeability are considered.

## NUMERICAL PROCEDURE

A computer program featuring the above defined plain-strain state has been developed. The heat transmission has been described by generalized diffusion formula:

$$Q+(\Lambda_i\theta,_i),_i+C_w\rho_w(k_iH,_i\theta,_i+k_i,_iH,_i\theta+k_iH,_{ii}\theta)+$$
$$+(C_w/g)(k_iu,_i\theta,_i+k_i,_iu,_i\theta+k_iu,_{ii}\theta)=C_E\dot{\theta} \qquad (1)$$

Equation (1) is valid only for isotropic permeability k of the soil and for the coefficient of thermal conductivity $\Lambda$. The sink of the heat is denoted by Q, the temperature of the soil by $\theta$, the specific heat of the pore water and of the soil by $C_w$ and $C_E$, density by $\rho_w$, the hydrostatic head by H, the excess pore-presure by u and the time derivative of the temperature by $\dot{\theta}$.

The continuity equation of water seepage has been derived in the following form:

$$(n\alpha_w+(1-n)\alpha_s)\dot{\theta}+(n(K_s^{-1}-K_w^{-1})-K_s^{-1}-E_{oed}^{-1})\dot{u}+E_{oed}^{-1}\dot{\sigma}+$$
$$+k_i,_i(H,_i+u,_i/\gamma_w)+k_i(H,_{ii}+u,_{ii}\gamma_w^{-1})=0 \qquad (2)$$

The coefficients of linear dilatation of water and soil are donoted by $\alpha_w$ and $\alpha_s$, $K_w$ and $K_s$ the bulk moduli and the time derivative of the effective stresses by $\dot{\sigma}$.

The stress-strain relations in the soil are described by the following relations:

$$\sigma_{ji},_j-f_i=\sigma_{ji},_j+u,_i-f_i=0 \qquad (3)$$

$$\dot{\sigma}_{ij}=2\mu\varepsilon_{ij}+\Delta\delta_{ij}\varepsilon_{kk} \qquad (4)$$

where $f_i$ denotes the components of volume forces in the soil. Kronecker's symbols are denoted by $\delta_{ij}$, $\mu$ and $\Delta$ are Lame's constants. The strain tensor components $\varepsilon_{ij}$ are given by the equation:

$$\varepsilon_{ij}=-(v_i,_j+v_j,_i)/2 \qquad (5)$$

The equations of heat transfer(1), behaviour of pore-presure(2) and stress-strain relations(3,4,5) can be treated simultaneously. A numerical procedure of the iterative-incremental type should be introduced. The time and the space are discretized by n intervals. For any r-th step the temperature state can be calculated if only the temperature state and the state of pore-pressure in the soil at the end of the preceding (r-1)th interval are known. When the temperature state in the r-th interval is known and when the state of pore pressure and deformation characteristics at the end of (r-1)th interval are given, the state of pore pressure in the r-th interval can be calculated. Now, the strain-state at the end of r-th interval can be deduced. The calculation of the remaining quantities at the end of this interval follows. The algorithm can then be repeated iteratively.

## APPLICATION TO TUNNELING

Artificial ground freezing is used as a structural support system as well as a water barrier in tunneling. The present analysis applies to a tunnel situated in a permeable soil below a water table with a close ring arrangement as shown in Fig.1. The horizontal freezepipes are used.

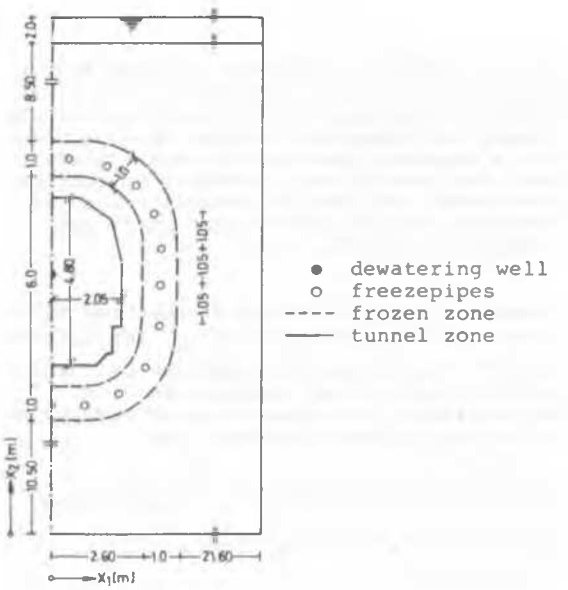


Figure 1. The freezing scheme using horizontal freezepipes

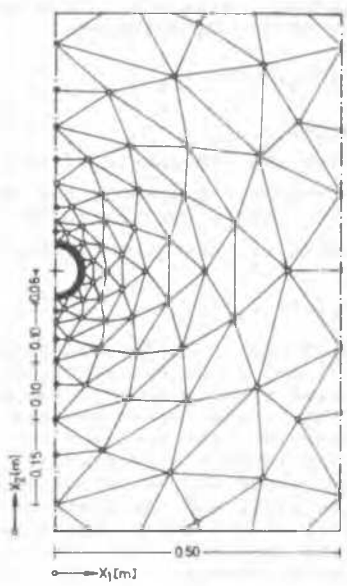

Figure 2. A symmetrical section of the FE mesh for the freezepipe system

In the first step, the transmission of heat and its influence on the time dependent state of the pore water pressure and on the stress-strain state in the soil is analysed on a micromodel which consists of a set of heat sinks on a horizontal line. Fig.2 shows a part of the triangular element mesh of that micromodel.

The boundary conditions in this model are as follows:

-each sink has a constant heat charge of -300J/ms
-the lower boundary of the whole region is at the distance of 11m from the sinks and at the constant temperature of $10^{0}$C; this boundary is impermeable and fixed,
-the upper boundary of the region is at the distance of 12m from the sink and is permeable.

The calculations have been carried out for three types of soil: sand, silt and clay.

In this paper the whole analysis is given for the silt only which has the following characteristics:

Unfrozen soil: unit weight of solid particles $\gamma_s$=27kN/m$^3$, the initial void ratio of soil e=0,7, modulus $E_{oed}$=a+bp´ with the following parameter values: a=500kPa, b=40. The coefficient of permeability k=10$^{-7}$ms$^{-1}$, degree of saturation $S_r$=100%, $\nu$=0,3, cohesion and angle of internal friction c=35kPa, $\phi$=25$^{0}$. The thermal characteristics of unfrozen soil are $\lambda$=1,418J/ms$^{0}$C and $C_E$=2,99MJ/m$^3$. (All above determined values have been stated on the ground of experiments, carried out in the laboratory of the University of Maribor).
The characteristics for the frozen soil are chosen from the literature. They are: $\nu$=0,49 and E=1,2.10$^7$kPa, $\lambda$=1,951J/ms$^0$C, the latent heat of the ground water and of the frozen soil are $C_L$=126,005MJ/m$^3$ and $C_E$=2,057MJ/m$^3$.

Fig.3 shows the freezing patern of the soil around the freezepipes. The lines describing the frozen regions refer to days. Fig.4 shows the isotherms after 10 days of freezing.

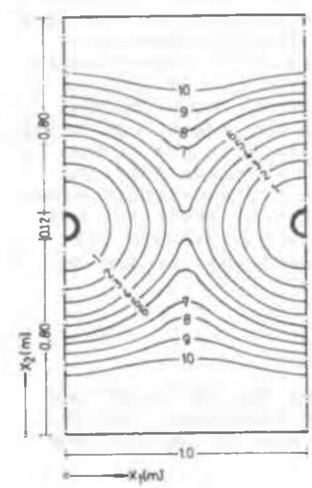

Figure 3. Progress of freezing in days

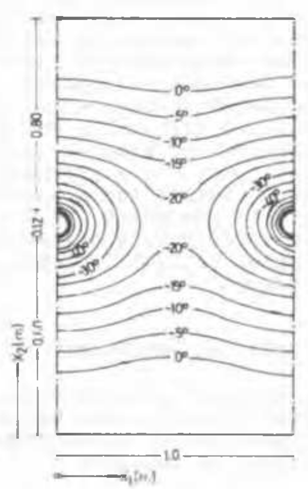

Figure 4. Isoterms after 10 days of freezing

When applying the same time rate of freezing as by sand and silt, the analysis of freezing of clays shows a great increase of pore pressures (Fig.5) and local plastification. Fig.6 shows these local failures in terms of safety coefficients, defined by the quotient $\tau_f/\tau$, $\tau$ being the actual shear stress in the soil and $\tau_f$ its failure value.

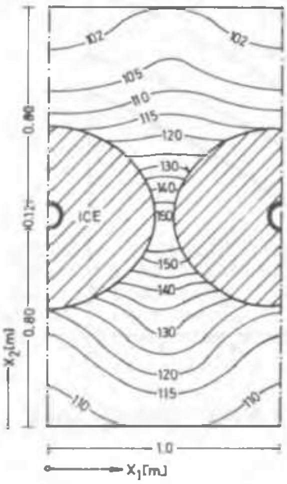


Figure 5. Isolines of excess pore pressures in clays 5 days after freezing (kPa)

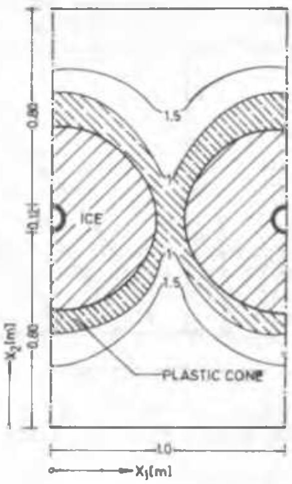


Figure 6. Isolines $\tau_f/\tau$ in clays 5 days after freezing

The process of freezing of the whole frozen zone around the tunnel as shown in Fig.1 has been carried out considering the following boundary conditions: in the middle, there is a dewatering well, the soil temperature is 10$^0$C, the power of the freezing system is 300J/ms, temperature of freezepipes is -50$^0$C. As to the geometry of the model (Fig.1) the upper boundary of the analysed region is free and situated 2m under the water table. The lower boundary is impermeable and fixed. The results shown refer to the silt the characteristics of which were given above.

Fig.7 shows the progress of freezing in days. The isolines of excess pore pressures after the freezing process are shown in Fig.8. The isolines of effective stresses $\sigma_{x_1}$ (horizontal), $\sigma_{x_2}$ (vertical) and shear stresses $\sigma_{x_1x_2}$ referred to the

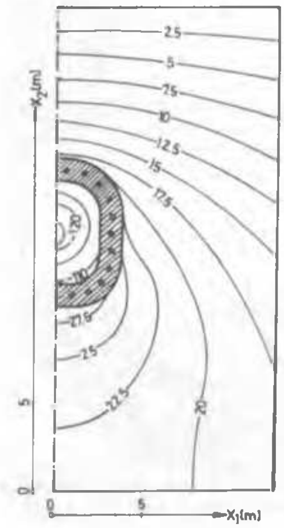


Figure 8. Excess pore-pressure isolines after freezing (kPa)

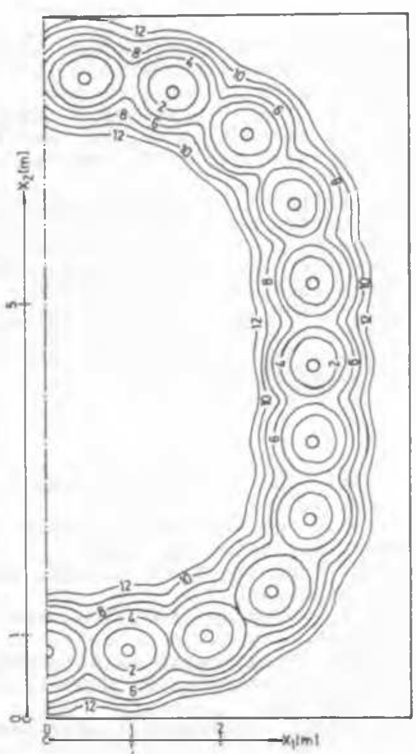


Figure 7. Progress of freezing in days

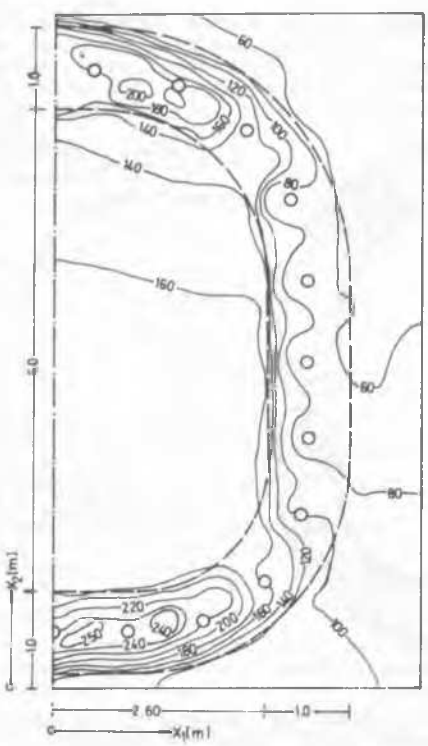


Figure 9. Isolines of horizontal stresses (kPa)

state of final freezing when thickness of the frozen zone is 1m have been determined. The isolines of horizontal stresses are shown on Fig.9 and the isolines of shear stresses are shown on Fig.10.

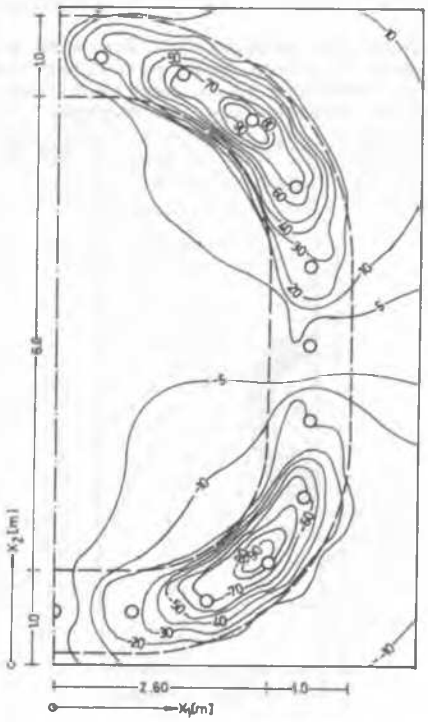

Figure 10. Isolines of shear stresses (kPa)

The analysis of the frozen zone after the excavation of the tunnel has been based on the FINEL computer program (Bedenik,1987) using the isoparametric finite elements. The shear stress state is shown in Fig.11. Fig.12 shows deformations of the frozen retaining zone after excavation of the tunnel.

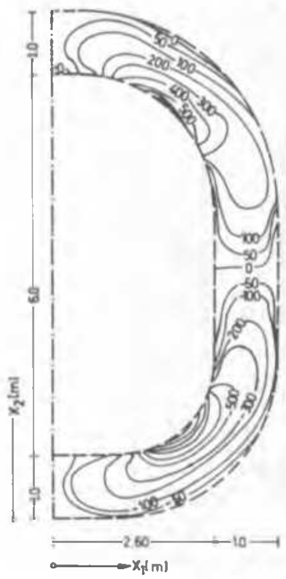

Figure 11. Isolines of shear stresses (kPa)

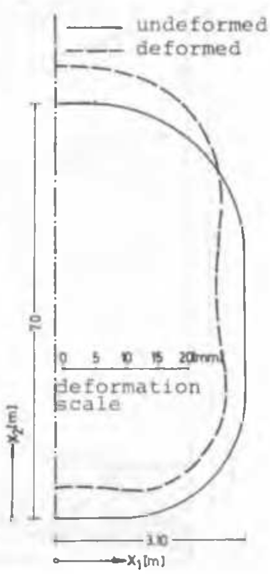


Figure 12. Deformation of the axis of the frozen zone

The comparison of the progress of the frozen zone in three different soils is shown in Fig.13.

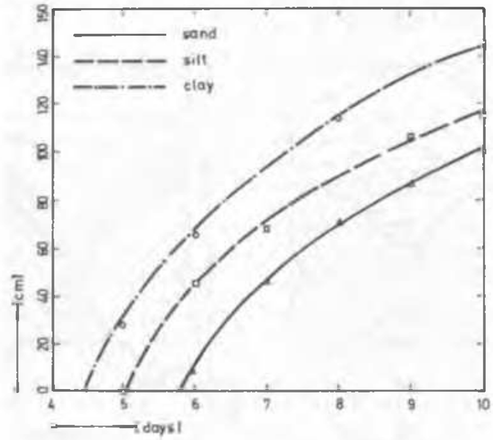


Figure 13. Progress of the frozen zone in days for sand, silt and clay

## CONCLUSIONS

The simultaneous numerical solution of equilibrium equations and of diffusion equations for heat transfer and fluid flow has been adapted for the analysis of the progress of freezing in tunneling and the corresponding effective stress change in the surrounding soil. As frozen and unfrozen soils are treated as hypoelastic bodies with Coulomb-Mohr's failure envelope, the stress strain analysis yields simultaneously, by safety coefficient $\tau_f/\tau$, the degree of the strength mobilization.

In the present version the computer program elaborated on the above fundamentals is limited to plane-strain states in the homogeneous, isotropic half-space. The stress-strain states appearing after the excavation of the tunnel, have been computed by using the FINEL computer program.

Favourable results of the application of the two computer programs obtained so far encourage further attempts to enlarge the applicability of the solution for cases with stress-dependent anisotropic permeability, of unstationary temperature field in the soil, of heterogeneous composition of soils, for cases of high deformability when the assumption of small strain ceases to be justified,etc.

## REFERENCES

Škrabl, S. (1988). Master's degree work. University of Maribor, Maribor.
Cook, R.D. (1981). Concepts and applications of finite element analysis. New York.
US Department of Tranportation. (1986). Soil Freezing. Washington D.C.
Bedenik, B. (1987). System FINEL. University of Maribor, Maribor.
Šuklje, L. (1980). Rheological relationships for soils. Research report, 1-128. University of Ljubljana, Ljubljana.
Battelino, D. (1988). Geofreezing, Research report, 1-86. University of Ljubljana, Ljubljana.